International Conference on Case Histories in Geotechnical Engineering

(2013) - Seventh International Conference on Case Histories in Geotechnical Engineering

02 May 2013, 4:00 pm - 6:00 pm

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Commemorate the Legacy of Ralph B. Peck
Seventh International Conference on Case Histories in Geotechnical Engineering
April 29 - May 4, 2013 CHICAGO
and Symposium in Honor of Clyde Baker

DAMAGE ON A MAIN GAS PIPELINE DUE TO LANDSLIDE OF SOILS DERIVED FROM VOLCANIC ASHES IN COLOMBIA

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ABSTRACT

In December 2011 a great proportion slope failed affecting the main gas pipeline (20" diameter) that supplies Colombia's southwest area. The landslide involved materials formed by soils derived from volcanic ashes and residual soils, typical in Colombia's central zone, which acquired great energy flowing in an avalanche through the natural watercourse. The avalanche generated soil erosion over the torrent banks, generating the watercourse deepening and the deterioration of adjacent slopes over which the pipeline runs.

There were identified as landslides triggering aspects the atypical raining period due to "La Niña" climatic phenomenon and the soils removal at the slope base due to a minor stream.

The article contains an analysis based on the characterization of this particular soil and the main landslide and avalanche incidence on the adjacent slopes stability. Additionally, it presents the results of the numerical soil-pipe interaction analysis with finite elements models that was completed to define and design the stabilization works.

INTRODUCTION

In 2011 an atypical rainy period which generated a national emergency with grave damage in highway infrastructure and public services occurred in Colombia and other South American countries.

One of the gravest damage was the one suffered by the main gas pipeline Mariquita – Cali (operated by TransGas de Occidente S.A. (TDO)) at PK-35, in the vicinity of Fresno and Padua municipalities (Tolima department). On 15 December 2010, a landslide with a volume higher than 1000m^3 dragged the 20"gas pipeline, generating its rupture and subsequent suspension of gas supply for nearly 5 million users at the Colombian west area.

The landslide is closely related to "La Niña" phenomenon. This phenomenon is usually associated to strong rain and floods in South America and Asia-Pacific and droughts in Africa. "La Niña" last event was registered between the end of 2011 and the beginning of 2012 according to the World Meteorological Organization (WMO). The Colombian Hydrology, Meteorology and Environmental Studies Institute (IDEAM) reported a 100% rain excess in the zone compared to the historical record.

The landslide displaced an important amount of soils derived from volcanic ashes located above residual soils and fractured and medially weathered clayey rocks (saprolite).

The main features on the case are presented below, including the definition of the landslide cause and the considerations on the design of the geotechnical works needed in order to guarantee the pipeline stability.

LANDSLIDE DESCRIPTION

From a geomorphologic point of view, the slide zone presents an abrupt topography, with high inclination and slopes dissected by intermittent drainages. The landslide, of translational type, involved soils derived from volcanic ashes sliding over residual soils and clayey rock saprolite. The movement presents strike south-north, 65m length, 60m width, and thicknesses that reach 15m, involving a volume of nearly 1000 m^3.

The remaining escarpment, after the landslide occurrence, is 30m high and has an average inclination of 60°. The estimated velocity of the displaced mass was of 17 m/s, corresponding to very rapid according to the scale proposed by Varnes (1978). The unstable mass acquired high energy due to the topographic inclination, the failure surface inclination and the high water content of the displaced soils. These soils present a cemented macrostructure which allow them to accumulate high water contents. After the landslide occurrence, the soil structure modified by remolding, lowering its shear strength and flowing at high speed. The initial movement, generated the gas pipe rupture which alignment east-west crosses the middle unstable slope in a perpendicular direction.

Afterwards, the displaced soil mass acquired avalanche features, moving through a natural water course and undermining the soils on the torrent bank, specially the one on the left due to its curved alignment. On the external bank of the curve, the flow reached an approximately height of 15m. The avalanched finally arrived at the pipe underwater crossing, located at 230m downstream from the rupture point, without affecting the pipe at this point.

As contingency measures to reestablish the gas supply, there were built two metallic structures in each flank of the landslide in order to hold the pipeline with steel cables.

The landslide location and the avalanche trajectory are showed in Fig. 1.

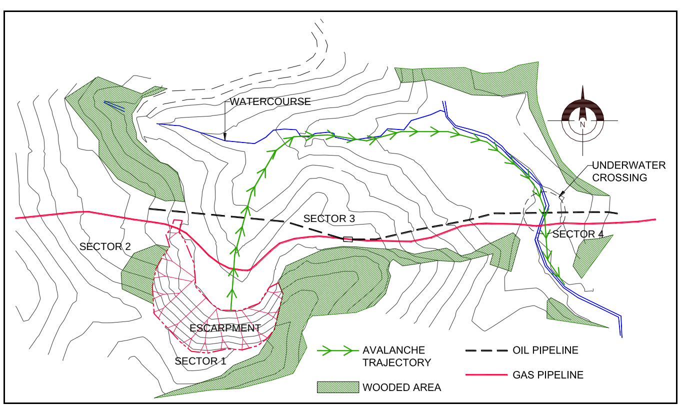

Fig 1. Sector and main features location

The event coincided with an especially intense rainy period caused by “La Niña” phenomenon; consequently the humidity increase in the soil due to intense rain was established as the main triggering factor. In this type of soil, the shear strength is highly dependent on cementation and suction under partially saturated conditions. The water content increase causes a reduction in suction and in consequence, an effective strength reduction. For the water content range established for the project soils, close to the liquid limit, there is an important reduction in suction with small increments in water content due to the high sensitivity of capillarity in the micro pores related to water content (Lizcano, A. et al., 2006). Additionally, as a secondary triggering factor there was identify the undermining on the slope base due a watercourse flow.

The avalanche consequences caused the need of geotechnical evaluation in the following sectors, whose localization is presented in Fig. 1:

Sector 1. Principal escarpment- directly affected by the landslide. There were evaluated the landslide triggering factors, the long term remaining stability and the possibility of damage of the pipeline air crossing and underwater crossing.

Sector 2. Western slope where the pipeline runs longitudinally in which is located one of the foundation structures for the air crossing built as contingency measurement. The stream removed the soils located on the base, reactivating a former instability processes. The new movements were evident due to tension cracks and displacements to occur on the order of centimeters.

Sector 3. Eastern slope from the affected area, where there is located the second foundation structure for the air crossing built as contingency measurement.

Sector 4. Gas pipeline underwater crossing, located downstream the landslide position. Quantification of the pipeline hazard level for possible displacement of the soils deposited along the watercourse (landslide remaining) or soils produced by a reactivation of the main escarpment.

The geotechnical analysis performed for sectors 1, 2 y 4 which represent the more interesting areas are presented below.

SECTOR 1 GEOTECHNICAL EVALUATION

This sector corresponds morphologically to a narrow top of a hill, which north front slide. The stratification, shown in Fig. 2, is formed superficially by soils derived from volcanic ashes, differentiated along depth due to their grain size variation. This grain size variation is attributed to different volcanic eruption events and later deposition spaced along time, as well as the weathering level increment with depth (Lizcano, A. et al., 2006). These soils are described as pale yellow clayey silts of volcanic origin with soft to firm consistency (Stratum

1) transforming with depth into yellow sandy silts of volcanic origin with soft to firm consistency (Stratum 2). Subsequently, there are found sandy silts of residual origin, dark yellow with oxide veins with medium to high density (Stratum 3) as transition to saprolite, weathered claystone in sandy-silty matrix with high density (Stratum 4). The oxide veins could indicate water table seasonal fluctuations.

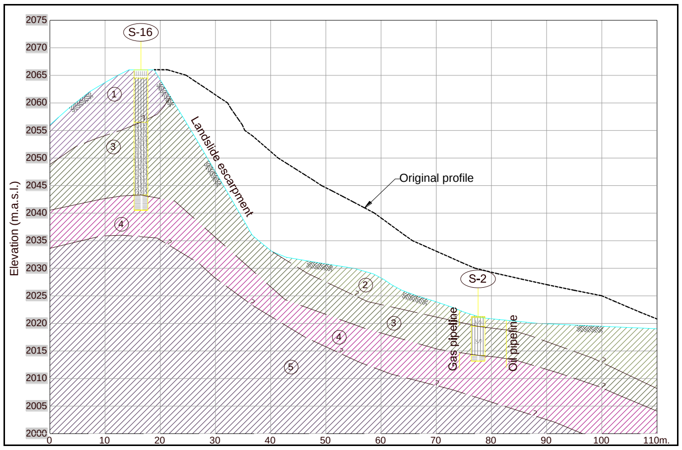

Fig 2. Sector 1 stratification

Figure 2 shows the original slope profile and failure surface geometry corresponding to the remaining escarpment after the landslide. A stability back analysis based on the two geometries described previously, the laboratory tests for the determination of the initial shear strength parameters and a superficial water level hypothesis (valid for a rainy period) was performed. In the calculation, there was considered the lateral undermining of the slope base due to the existing water course. With these calculations there were adjusted the shear strength parameters (φ, c) of the soils derived from volcanic ashes in order to obtain a factor of safety F.S.=1, with which failure was simulated.

The main geomechanical parameters of the described stratums are included in Table 1.

Table 1. Geotechnical Residual Parameters

Stratum	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Friction angle (°)
1	17	14	37
2	14	36	33
3	18	25	36
4	19	49	38

After the back analysis and aiming to evaluate the slope stability under current conditions, there were performed stability calculations under static and pseudo-static conditions. The results show acceptable stability under static conditions (F.S. = 1.23) and marginal stability under pseudo-static conditions (F.S. = 0.94). In order to increase the factor of safety, the slope intervention could include a morphological reconfiguration (terracing) to acquire the reduction of mobilizing forces.

SECTOR 2 GEOTECHNICAL EVALUATION

For this sector analysis there was established the stratification shown in Fig. 3. The installation of Casagrande piezometers for the constant monitoring during several months allowed to conclude the inexistence of a defined water table in the soils derived from volcanic ashes and the existence of piezometric levels produced by artesian water in the inferior stratums formed by saprolite. This conclusion was verified during the horizontal drains and anchors boring execution, in which the operator reported the inexistence of free water in the soil and water rising through the boring pipe once the saprolite stratum was reached.

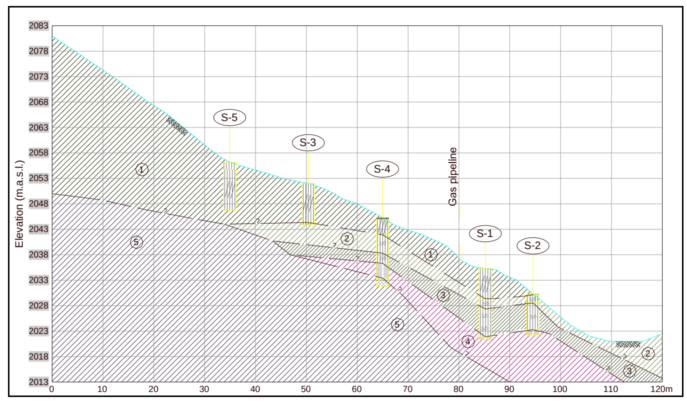

Fig. 3. Sector 2 stratification

As it was mentioned before, the material flow produced by the main landslide removed the soils located on the base of the sector 2 slope, reactivating a former instability processes evidenced by displacements on the order of centimeters and superficial tension cracks. To determine the failure surface depth location there were employed the standard penetration test (SPT) results to identify the rigidity and resistance main variations (Soils derived from volcanic ashes contact with residual soils and saprolites) (Fig. 4). On the surface, the failure geometry was limited by the cracks observed on field.

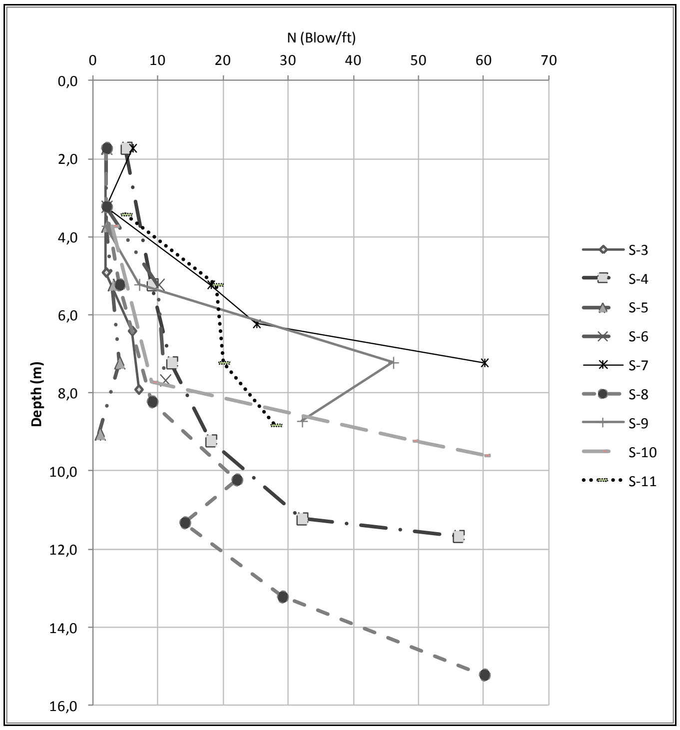

Fig. 4. Standard penetration test (SPT) results

In order to follow the variations of displacements on the slope and to implement an alarm system related to intolerable strains on the pipe there were installed three inclinometers along the pipeline alignment. During the geotechnical study development, the measured displacements were inferior to 1cm. Even though the displacements were small, the stability calculations were calibrated based on this information, facilitating the location of the failure surface.

Limit equilibrium analysis

Through calculations with the limit equilibrium method and using a failure surface defined as it was described previously, the strength parameters (φ, c) were adjusted with back calculation once again. Congruent with the displacements reported during rainy season (under 5cm), the parameters were adjusted in order to obtain a factor of safety close to F.S.=1.15. This factor of safety, typical during intense raining season is inadmissible for the pipeline function. For this reason there was evaluated the efficacy of lowering the piezometric pressure with horizontal drains. The definition of the lowering piezometric line was obtained employing a flow model with the finite elements method using the program Slide 5.1. The new stability calculation considering the horizontal drains effect resulted in a factor of safety of F.S.=1.94 which is acceptable for the pipeline operation. The efficacy of the drainage system after its construction has been confirmed with piezometric level (piezometers) and displacement measurements (inclinometers).

Pipeline stress-strain analysis

This analysis was carried out with the objective of establish the current stress applied on the pipeline, due to the slope displacements aided by the main landslide, and compare it with the admissible pipeline stress. Furthermore the analysis is performed to quantify the stress on the pipeline caused by the horizontal drains construction. To achieve these objectives it was performed a stress-strain numerical model with the finite element method using the software Plaxis V8.1.

The shear strength parameters (φ, c) were obtained by means of back calculation using the limit equilibrium method described previously.

The elasticity modulus values were defined based on the shear modulus under low strains obtained from the direct shear test performed on representative samples, empirical correlations with standard penetration test (SPT) results (DIN Norm 4094) and back calculations adjustments with the software Plaxis V8.1 based on the displacement measurements obtained on field. The elastic modulus obtained from the different methods produced similar results.

The permeability coefficient values were assigned according to reference information and the experience of C.I.C. with soils of similar features. The employed parameters for the main stratums are shown in Table 2.

Table 2. Geomechanical parameters strength-strain model

Stratum	Unit weight (kN/m^3)	Cohesion (kN/m^2)	Friction angle (°)	Elastic Modulus (kN/m^2)
2	17	5	30	5000
3	17	5	37	6000
4	19,5	35	36	8000

With the parameters presented in Table 2 and the piezometric level near to the surface (rainy season) there were obtain displacements on the order of 2cm (Fig. 5), mainly on the horizontal direction. The pipeline reaches displacements on the same order of magnitude. The later piezometric level lowering (horizontal drains) induces total additional displacements on the order of 1.5cm.

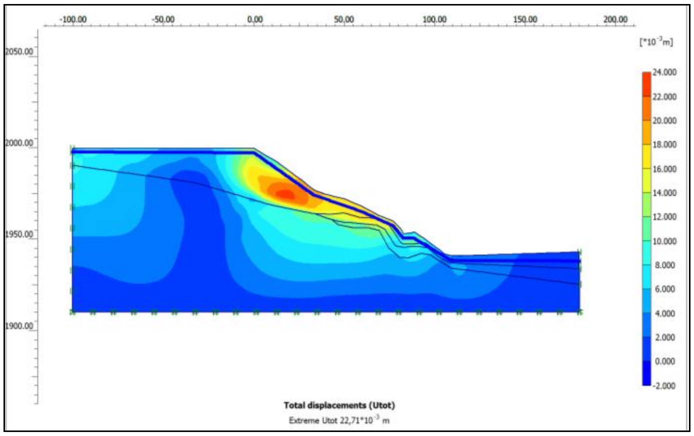

Fig 5. Total displacement (superficial piezometric level)

According to the results, the maximum momentums on the pipe are located at the pipeline change of direction, being vulnerable points. The maximum registered momentum is 45 KN-m/m.

Table 3 contains a summary of the obtained results related to the stresses and displacements on the pipeline.

Table 3. Numerical Model results (Pipeline)

	Superficial piezometric Level	Piezometric Level lowering	Displacement variation and stresses on the pipeline (%)
Maximum soil displacement (cm)	2,3	2.0	13
Maximum pipeline displacement (cm)	2.0	1,4	30
Maximum axial force pipeline (kN/m)	146,5	60,6	58,6
Maximum shear stress pipeline (kN/m)	39,7	20,7	47,7
Maximum momentum pipeline (kN-m/m)	93,0	45,3	51,4

The construction of horizontal drains induced a clear distress of the pipe. A verification of axial force under tension, shear strength and flexion on the pipeline was performed, comparing the strength admissible values for metallic elements according to the Colombian Seismoresistant Construction Regulations (NSR-10). From this verification there were obtained factors of safety higher than F.S.=10, being the pipeline flexion factor of safety the lowest. According to the analysis the factors of safety are admissible.

GEOTECHNICAL EVALUATION SECTOR 4

In sector 4 it was evaluated the possibility that the slide mass accumulated on the water stream displaced as an avalanche, generating undermining at the underwater crossing and putting at risk the pipeline located at nearly 3.5m depth.

Initially, it was estimated an approximate avalanche height, which involves the volume of accumulated material placed on the water stream and the inference of a deposit area nearby the underwater crossing.

Afterwards it was calculated the avalanche caudal. This calculation has as input data the sliding mass velocity. The velocity was obtained adjusting the Voellmy y Perla (1980) equation for snow avalanches in alpine zones.

The following step was the undermining calculation due to the sliding mass using the Lischtvan-Levediev method for course and cohesive soils, since the slide material is between these two types of soils. The undermining calculations results are on the order of 1m. This undermining magnitude represents a low hazard for the gas pipeline given that the pipe would be cover by a soil layer of nearly 2.5m thick at this point.

CONCLUSIONS

The following conclusions were obtained based on the accomplished study:

The superficial soils found in the area and involved in the landslide are derived from volcanic ashes. These soils present a special behavior due to the cementation that provides them a porous macrostructure (high void ratio) capable of holding great amount of water. At the formation base there are found saprolites and medially weathered clayey rock.

The piezometric level is originated by the artesian water pressure trapped at saprolite level due to the water recharge uphill.

The landslide was triggered by the atypical rainy period “La Niña” which increased the water content in the soils derived from volcanic ashes at the contact with saprolites, later becoming the failure surface. The water content increase drastically decreased the suction and the shear strength supporting the landslide occurrence. The removal of the soil located on the slope base by an existing water course was identified as a secondary triggering factor.

The landslide displaced nearly 1000m^3 of soil, which flowed as an avalanche through a water course with an estimated velocity of 17 m/s. The flow undermined the watercourse banks, reactivating a former landslide on the western slope were the pipeline runs longitudinally.

The stability analysis indicated that in order to guaranty adequate factors of safety under static and pseudo static conditions, and the pipeline stability, it is necessary to morphologically reshape the main escarpment, to build horizontal drains on the western and eastern slope (where there are located the foundation for the air crossing along the main landslide). As a contingency measurement for the pipeline protection, once the landslide occurred, there were constructed anchors on the western slope.

The western slope analysis with the finite element method showed that the current stresses and strains are still under adequate safety margins.

Monitoring during some months after the landslide event with piezometers and inclinometers confirmed the efficacy of the adopted measurements.

The underwater crossing located downstream from the main landslide area presents a low hazard level related to future eventual avalanches.

ACKNOWLEDGEMENTS

The authors wish to thank TransGas de Occidente S.A. (TDO), especially Eng. Alvaro Vega, for authorizing the use and publication of the information related to the event described above.

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